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S. Sambasiva Rao

R. G. Beale

M. H. R. Godley

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S Sambasiva Rao[1], R G Beale[2] and M H R Godley[3]

Abstract

Pallet racks, often fabricated using cold-formed steel, are used for the storage of goods. Uprights of these racks are braced in the cross-aisle direction forming a frame, which behaves like a built-up column. Evaluation of the shear stiffness of this frame is needed to determine the buckling load. Currently two approaches prevail in the rack industry to determine the shear stiffness. The RMI code uses a theoretical formula and the FEM code requires testing. There is a considerable difference in the stiffness values determined by two approaches. The present paper describes experimental and numerical studies conducted at Oxford Brookes University to evaluate shear stiffness in an ongoing research project.

Introduction

Pallet racks, often fabricated using cold-formed steel, improve the storage of goods by the efficient use of the cubic space available for storage. The uprights (columns) of these racks are braced in the cross-aisle direction as shown in Fig. 1, forming a frame, which behaves like a built-up column. In recent years, large pallet racking systems have been used containing tall and narrow upright frames. Such frames need to be checked for stability in the cross-aisle direction to

[1] Research Student, School of Built Environment, Oxford Brookes University, Oxford, UK
[2] Principal Lecturer, School of Technology, Oxford Brookes University, Oxford, UK
[3] Senior Research Fellow, School of Built Environment, Oxford Brookes University, Oxford, UK

prevent any potential collapse, which could lead to loss of human life. The stability of such frames in the cross-aisle direction i.e. about the z-z axis [see Fig. 2] depends on

- Overall behaviour of the frame. This is affected by elastic flexural buckling and shear deformations,
- Local thin walled behaviour of each upright,
- Forces in diagonal bracing members due to eccentric joints and
- The bolt slip due to the bolt-hole clearance, if the connection is bolted.

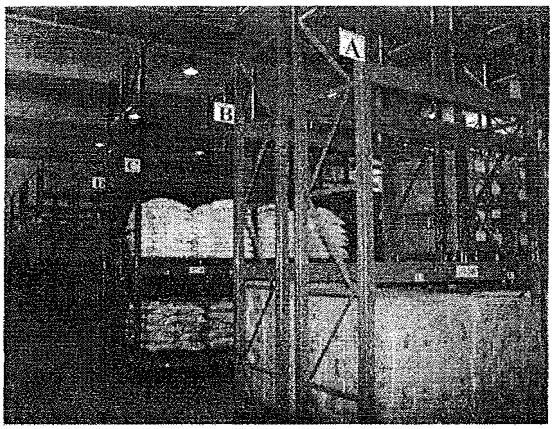


Fig. 1: A Rack Structure

Flexural deformations can be calculated using established theories. However, there is only limited research available to find the shear deformations, particularly in cold-formed built-up columns. Accurate evaluation of these shear deformations is needed to determine the elastic buckling load and sway deflections of upright frames. Hence, research is being undertaken to predict the actual shear stiffness values of upright frames.

The present paper reviews the published literature and existing design approaches for evaluating shear stiffness values of built-up columns or pallet rack upright frames. It also describes the experimental and numerical studies to evaluate shear stiffness of upright frames. A simple frame analysis has been carried out on upright frames using the finite element analysis (FEA) program, LUSAS. The effects of various factors such as the flexibility of uprights, eccentric loading on bracing members, the aspect ratio (panel length/panel depth) of frame panels and bolt bending, on the shear stiffness of upright frames are studied. The study is limited to upright frames with bolted lacings or battens, which are common in Europe. In future, more refined FEA models that will be validated against test specimens can be used to generate additional data to obtain a better method of evaluating shear stiffness of upright frames.

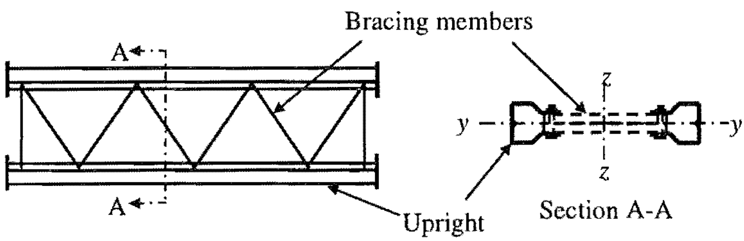


Fig. 2: A Typical Upright Frame

State of the art

Literature review

The first theoretical investigation into the affect of shear on compressive strength of columns was by Engesser (1891), who modified the Euler analysis for axially loaded columns to account for shear [Galambos 1988]. Timoshenko (1949, 1961) was first to include the affect of shear in built-up columns. He proposed a study of shear effects in built-up laced and battened members. In 1952, Bleich extended Engesser's work to the critical load analysis of built-up columns. Shear effects in battened and laced members were more recently studied by Lin, Glauser and Johnston (1970). They recommended shear formulae incorporating the effects of stiffened zones at the ends of built-up members, eccentricities in the joints, and net span of chord between batten plates. Gjelsvik (1990, 1991) reviewed the different methods for evaluating shear stiffness of built-up columns.

A review of literature indicates that Timoshenko's theory is widely used for evaluating the shear stiffness. He assumed that a built-up column has a large number of panels and hence an equation, derived to account for shear in solid columns, is used for the prediction of critical loads. Shear deformations are attributed to the deformability of lacing bars or battens. Shear stiffness equations are derived for different types of laced or battened patterns. A representative equation for evaluation of shear stiffness, S, (for a single lacing pattern) is shown below:

$$\frac{1}{S} = \frac{1}{A_d E \sin\phi \cos^2\phi} \tag{1}$$

where, E – Young's modulus of the lacing member
A_d – Cross-sectional area of diagonal lacing members
ϕ – Angle of inclination of lacing member with the line drawn perpendicular to upright

The above approach was derived for hot-rolled built-up columns and is not directly related to pallet rack structures. Very few studies have been conducted on cold-formed steel built-up columns to check its validity. Djafour, Megnounif and Kedral (1999) analysed elastic stability of cold-formed built-up columns using the finite strip method and found it can predict buckling loads accurately. Chwan (2001) under the supervision of Beale and Godley carried out shear tests on pallet rack upright frames at Oxford Brookes University. Dubina et al (2002) carried out a numerical study on behaviour of built-up columns made of cold-formed C-sections connected with bolted C-stitches, and validated design approach proposed by Rondal and Niazi (1990, 1993) for determining buckling strength of battened cold-formed built-up columns. Their study was not extended to pallet rack upright frames.

Design practice in the rack industry

Currently there are two approaches prevailing in the rack industry to consider the effect of shear. One approach is to use a theoretical formula based on Timoshenko, and the other is to determine the shear by testing. The codes considered for the review in the present study are Storage Equipment Manufacturers' Association (SEMA) code, Rack Manufacturers' Institute (RMI) code, Federation Europeenne de la Manutention (FEM) code and the Australian code (AS4084-1993).

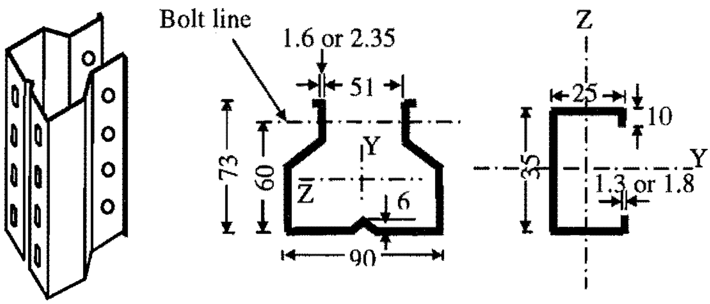
The SEMA code is the only code where shear in upright frames is not considered. The RMI and Australian codes recommend a theoretical approach based on Timoshenko (Equation 1). The FEM code adopts testing to evaluate shear stiffness per unit length of the frame structure. The code requires the test sample to be a frame assembly with a number of bracing panels. The testing procedure recommends using a minimum of three panels in the case of laced upright frames and a whole number of panels in the case of battened frames. There is a considerable difference in the shear stiffness values determined by these two approaches. The test values based on the FEM code are sometimes 20 times lower than the theoretical values [Chwan 2001] calculated using the RMI code.

A review of the available literature and design recommendations in various rack industry codes was carried out. This indicates that substantial research was not carried out on cold-formed built-up columns to arrive at the appropriate design approach for the evaluation of shear. Hence, the present research was undertaken in which both experimental and numerical studies were carried out. The experimental program conducted at Oxford Brookes University for the evaluation of shear is described in the following section.

Experimental program

Test specimens

Tests were conducted on full sized upright frames. Uprights (columns) of the frames were open perforated lipped channels with additional bends and the bracing members were simple lipped channels. In total, 21 tests were performed (Chwan & the authors) by changing upright size, number of panels in the frame (2.5 or 3 panels), aspect ratio of the panel (panel length/depth varying from 1.14 to 3.23) and lacing pattern (channels back to back or front to front). Typical upright and bracing members are shown in Fig. 3. Note that all the dimensions mentioned in Fig. 3 are in mm.



(a) A view of Upright (b) Upright section (c) Brace section

Fig. 3: Typical upright and bracing members

In this paper, the upright with a 2.35 mm thickness is called the heavy upright and the upright with a 1.6 mm thickness is called the light upright. Though series 1 and series 3 uprights are light uprights, their cross-sectional properties are different, as there are small changes in their profiles. 1.3 mm thick bracing members were used in series 1 and 2 tests whereas 1.8 mm thick bracing members were used in series 3 tests.

Table 1 shows cross sectional properties of upright and bracing members that were used for testing. G_Y is the distance of the centroid of the upright from its back face centre line.

Table 1: Sectional properties of upright and bracing members

Member	Series	Net Area (mm^2)	Moment of Inertia (mm^4)		Centre of gravity (mm) G_Y	Torsion constant (mm^4)
			I_Y	I_Z		
Upright	1	252.3	457738	223195	24.47	348
	2	453.2	661683	321806	24.41	1090
	3	324.0	372205	163060	22.91	294
Bracing member	1 & 2	123.8	23658	11129	9.98	70
	3	167.1	30879	14307	9.73	180

Test Arrangement

The basic arrangement of the test upright frame can be seen in Fig. 4. The frame was placed in the horizontal plane between rollers, which coincides with the points of intersection of the bracing members. The positions of the rollers were adjusted so that the frame just fits snugly between them with no looseness. The roller condition at the nodes was achieved by putting two PVC sheets in between upright sections of the frame and the packing of the test rig. The test layout and arrangement of displacement transducers (LVDTs) are shown schematically in Fig. 5. One leg of the frame was pinned at one end so that it was prevented from moving horizontally, as at point A in Fig. 5. The load was applied along the centroid of the other leg, at point B in Fig. 5.

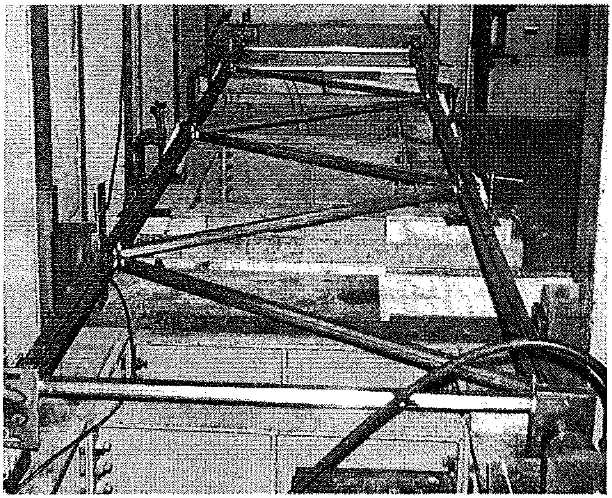


Fig. 4: Test upright frame in the laboratory

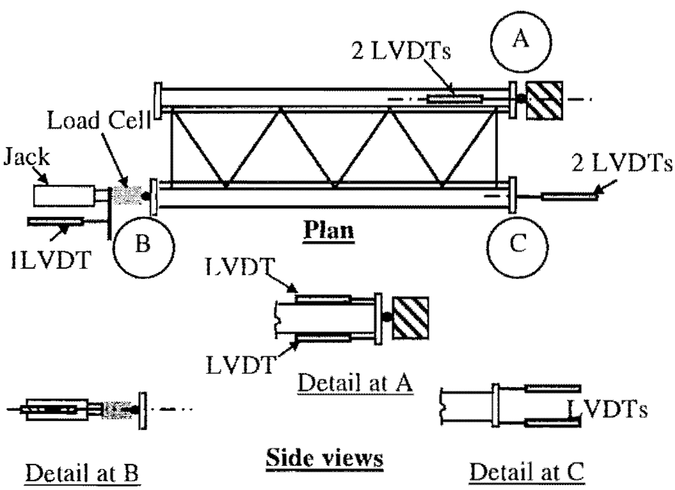


Fig. 5: Test set-up and LVDT locations

At point A, the frame was pinned in all three translational displacements. Two displacement transducers were also placed at A in the direction of the upright to determine any movement of the support. At point B, a load cell of 6 kN capacity was connected to a jack of 230 kN capacity and an LVDT was placed there to control the loading. Two LVDTs were placed at point C to measure the displacement of the loaded upright along its own axis. LVDTs were placed at bottom and top of the upright base plate. The mean value of the two LVDTs placed at C was considered for further calculation.

Loading was applied gradually using the jack with a load cell at the rate of 0.1 kN/sec. The readings from the LVDTs were recorded using a data acquisition system. The maximum load applied in the test was kept low (5 kN) so that, there was no visible damage on the specimens. After reaching the maximum load, the frames were unloaded to 0.5 kN. The frames were reloaded and unloaded between loads 0.5 kN and 5 kN for 5 to 6 cycles in each test. This was carried out to avoid any error in evaluating shear due to bolt slip.

Full-scale test results

After acquiring the data from the data acquisition system, the load applied on upright was plotted against the corresponding deformation to arrive at a load-deformation curve. A typical load-deformation curve for a tested frame is shown in Fig. 6.The slope, k_{ti}, was obtained by fitting a linear trend line to the cyclic loading applied in the test program omitting the first cycle. The first cycle results were not considered to avoid errors due to initial settlement of the joints.

Then, the transverse shear stiffness of the frame, S, was calculated by

$$S = \frac{k_{ti} D^2}{l} \qquad (2)$$

in which, l is the length of the frame, and D is the distance between the centroidal axes of the upright sections. This formula is in accordance with the FEM code.

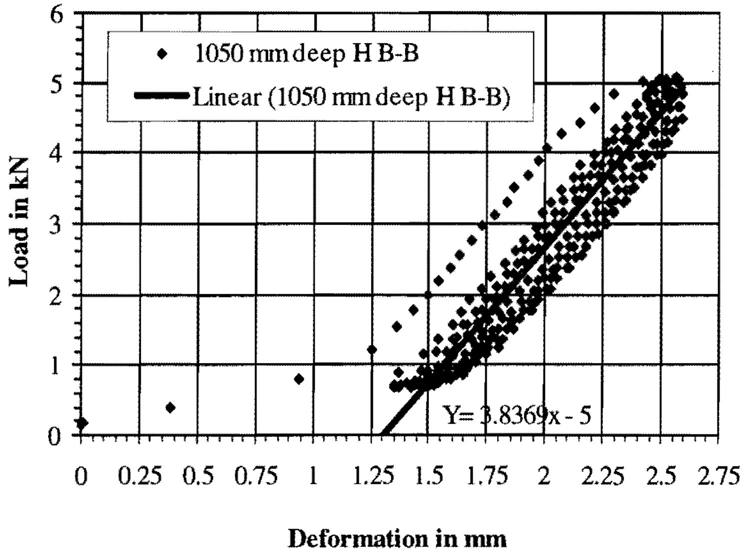


Fig. 6: A typical load-deformation curve

The graph shown in Fig. 6 is for an upright frame with back to back bracing pattern, heavy uprights and centre-to-centre depth of 1050 mm. In this case, the slope of trend line is 3.8369 and hence k_{ti} is 3.8369. After getting k_{ti} values from graphs, shear stiffness values can be easily determined using the equation 2. For the case shown in the graph, the length of the frame (l) was 3000 mm and the distance between the centroidal axes of the upright sections (D) was 1050 mm. Hence, the shear stiffness value for the case is 1413 kN. Similar types of graphs were drawn for all other tests and finally shear stiffness values were obtained and tabulated in Table. 2.

Table 2: Shear stiffness values determined by testing

Centre to centre distance between uprights, *D* (mm)	**Test values (kN) [FEM]**		**Timoshenko values (kN) [RMI]**	**[Theory/Test]**	
	Bracing pattern		Bracing pattern not considered	**Bracing pattern**	
	Back to Back	Lip to Lip		Back to Back	Lip to Lip
Series 1; Length of the frame (*l*) is 3000 mm and having 2.5 panels. Light (L) uprights					
1050	1391	634	9677	7.0	15.3
870	1015	447	9955	9.8	22.3
670	609	312	9579	15.7	30.8
520	514	319	8387	16.3	26.3
370	292	292	6068	20.8	20.8
Series 2; Length of the frame (*l*) is 3000 mm and having 2.5 panels. Heavy (H) uprights					
1050	1413	785	9677	6.8	12.3
870	1068	481	9955	9.3	20.7
670	-	403	9579	-	23.8
520	563	389	8387	14.9	21.6
370	-	296	6068	-	20.5
Series 3; Length of the frame (*l*) is 3600 mm and having 3 panels. Light (L) uprights					
1050	1881	-	14070	7.5	-
1050	1606	-	14070	8.8	-
1050	1937	-	14070	7.3	-

Based on the test results reported in Table 2, it is very clear that test results do not compare with theoretical values. At higher values of aspect ratios of the panel, the shear stiffness values differ significantly between theory and tests. The factors affecting the experimental results that are not considered in the theoretical formula [equation 1] are:

- The eccentricity induced due to bracing pattern, which has a major role to play in shear stiffness of pallet rack upright frames. The lip-to-lip to bracing pattern has more eccentricity in the connection and hence has a lower shear stiffness values compared to the back-to-back bracing pattern.
- The cross-sectional properties of the upright, which also contribute to the shear stiffness of the frame

Timoshenko's theory does not consider the above two factors whereas tests show they have a role to play in shear stiffness of frames. Hence, Timoshenko's theory should not be used for the evaluation of shear stiffness in upright frames. The affect of each factor is further studied using the finite element method.

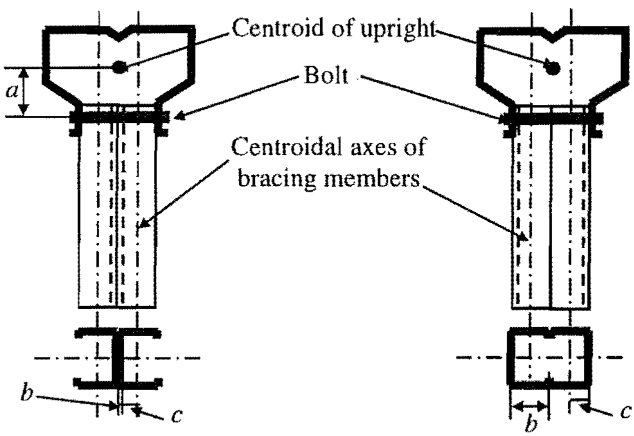
Numerical modelling

A linear analysis was carried out on upright frames using the LUSAS finite element software. For the purpose of illustration, the 1050 mm deep heavy upright frames that have been tested are presented here. Both back-to-back and lip-to-lip bracing pattern cases were studied. The elastic modulus of the steel was taken to be 209000 N/mm^2 in accordance with the manufacturer's specifications. The boundary conditions were kept the same as in the testing Loading was monotonic and bolt slip was not considered; hence these linear analysis results can be compared with the test results obtained from the trend line.

Initially the frame was modelled as a simple two-dimensional truss system, wherein both upright and bracing members were modelled using bar elements (BAR2). Results of this analysis were compared with hand calculation and results obtained by another frame analysis program, SAND, to check the validity of the FE model. Later upright frames were modelled as two-dimensional rigid and pin jointed frames. In the case of a rigid frame, both upright and bracing members were modelled using thin beam elements (BM3) i.e. shear in beams was neglected. In the pin jointed frame model uprights were analysed using thin beam elements and bracings with bar elements. In the case of above three models the joints were concentric. However, in practice, there is eccentricity in connections along with other factors such as bending in the bolt due to forces coming at the joint, rotational degree of freedom for the bracing members about bolt axis, etc. The modelling of these parameters is discussed below.

Connection eccentricities

The connection detail at a joint in upright frame can be seen in Fig. 7. In linear analysis of the frame, there are three eccentricities; (i) due to upright centroidal distance from bolt, *a*, (ii) due to actual force transfer between bracing and upright, *b*, and (iii) due to bracing centroidal distance from load transfer point, *c*.



(a) Back-to-back bracing pattern (b) Lip-to-lip bracing pattern

Fig. 7: Typical connection detail at joint

Eccentricities due to upright centroidal distance from bolt (*a*) and due to bracing centroidal distance from load transfer point (*c*) are same in both back-to-back and lip-to-lip bracing pattern cases. However, the eccentricity due to actual force transfer between bracing and upright (*b*) is different. It is negligible i.e. half the thickness of bracing member in the case of back-to-back braced frame and large in case of lip-to-lip braced frame [see Fig. 7]. Hence, a significant reduction in shear stiffness was seen in lip-to-lip braced frames. Thin beam elements were used to model all the three eccentricities in the FE model.

Bolt bending

In a frame with a single layer of bracing members, one bracing member is subjected to tension and the other will be subjected to compression at each joint as shown in Fig. 8. These force components induce bending in the bolt. The effect of this bolt bending in the FE model was achieved by modifying the stiffness of the thin beam element connecting upright and bolt. Bending in the bolt will be more predominant in back-to-back braced frames.

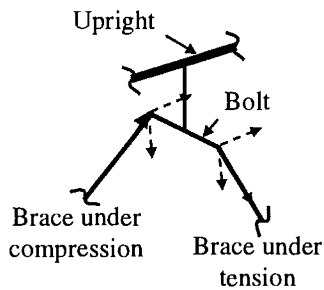


Fig. 8: Forces at a joint

Rotational degree of freedom about bolt axis

Bracing members in the uprights were connected using single bolts. Hence, the joint can be considered as a pin and bracing members are free to rotate about bolt axis. Three-dimensional joint elements (JSH4) available in the LUSAS were used to arrive at this condition in the FE model. These joint elements have three rotational and three translational degrees of freedom wherein stiffness values can be given to achieve the desired condition of connectivity. In addition, constraint equations can be used to enforce displacement restraints. In the present study, constraint equations were written for joint elements such that the rotational degree of freedom about the bolt axis was released and all other rotational and translational displacements were arrested. In Table 3, the shear stiffness values obtained using numerical analysis of all these models are reported.

Table 3: Comparison of numerical analysis results with theoretical (RMI) and test values

FE model	Back-to-back braced frame			Lip-to-lip braced frame		
	RMI (kN)	LUSAS (kN)	Test (kN)	RMI (kN)	LUSAS (kN)	Test (kN)
Truss	9677	8175	1413	9677	8175	785
Pin-jointed frame		8175			8175	
Rigid frame (A)		8240			8240	
(A) + all eccentricities (B)		5290			4265	
B + bolt bending (C)		4180			4040	
C + rotational release about bolt axis		3210			3850	
Note: Frames considered for illustration are 1050 mm deep heavy upright frames. Both back-to-back and lip-to-lip bracing pattern cases were presented.						

The results obtained for truss, pin-jointed frame and rigid frame in the FE analysis are 15% lower than the Timoshenko's theory (RMI). It is due to inclusion of axial and flexural stiffness of uprights in FEA model whereas Timoshenko's theory is independent of stiffness of uprights. In the models illustrated, the shear stiffness values from the numerical analysis are 2.3 times higher than the test values for back-to-back braced frame and 4.9 times the test values for lip-to-lip braced frame. From Table 3, it can also be noted that the effect of connection eccentricities is significant in the case of lip to lip bracing pattern case and the affect of bolt bending is more pronounced in back to back braced frames. Rotational release about the bolt axis has more effect on back-to-back braced frames. The current models do not consider all the effects and further study has to be carried out to find the significance of joint flexibility.

Conclusions

A review of literature indicates that there are two approaches prevailing in the rack industry to determine the shear stiffness of upright frames. The RMI code uses a formula based on Timoshenko's theory and the FEM code requires testing. There is a considerable difference in the stiffness values determined by the two approaches. Hence, research has been undertaken at Oxford Brookes University. Experimental studies were conducted and primitive FE models were developed using linear analysis. The effects of various parameters such as connection eccentricities, bolt bending and rotational release about bolt axis was identified. Further study needs to be carried out to find the significance of joint flexibility and to propose a better procedure for the evaluation of the shear stiffness of upright frames.

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